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RELIABILITY ASSESSMENT OF EXISTING CONCRETE BRIDGES

OCENA NIEZAWODNOŚCI ISTNIEJĄCYCH MOSTÓW BETONOWYCH

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Abstract: *The remaining working life of an existing reinforced concrete bridge is analysed considering the serviceability limit states of crack width. Two new models of crack width provided Eurocode EN 1992 and in the Model Code 2010 are applied. The probabilistic methods of structural reliability are used for assessing the crack width limits and remaining working life of existing concrete bridges. It appears that the initial reliability with respect to the serviceability requirements resulting from the original design of the bridge seems to satisfy the required target reliability level recommended in Eurocodes. However, the reliability index significantly decreases with the reduction of the reinforcement area due to corrosion.*

Keywords: *existing bridges, working life, failure probability, crack width*

Streszczenie: *Pozostały okres żywotności istniejącego żelbetowego mostu został poddany analizie, w której szerokość pęknięcia przyjęto za graniczne kryterium użyteczności. Wykorzystano dwa nowe modele pęknięcia, których dostarczyły Eurocode EN 1992 i Model Code 2010. Probabilistyczne metody niezawodności strukturalnej zastosowane do oceny granicznych szerokości pęknięcia i pozostałego okresu żywotności istniejących mostów ze zbrojonego betonu. Należy sądzić, że początkowa niezawodność w odniesieniu do wymagań użyteczności wynikających z oryginalnego projektu mostu wydaje się zadowalać wymagany docelowy poziom niezawodności zalecany przez Eurokody. Jednakże w wyniku korozji współczynnik niezawodności znacznie maleje wraz z redukcją strefy zbrojenia*

Słowa kluczowe: *istniejące mosty, okres żywotności, prawdopodobieństwo awarii, szerokość pęknięcia*

1. Introduction

An extended use of existing structures is of a great importance for many countries. It has significant economic, social and cultural impacts. Many buildings and bridges, built in the Czech Republic and in other European countries in the 1960s, are now reaching the end of their working life. They require assessment and rehabilitation to assure their further safety and economical exploitation.

The assessment of existing structures differs from procedures taken during the design of new structures and may require application of sophisticated methods. In many cases these methods are beyond the scope of common standards for structural design. The prescriptive documents cannot be directly applied for the assessment, as the actual state of structures and their materials must be taken into account. Moreover, the current standards have often more severe requirements than the codes applied at the time of original design. Although some existing structures appear to have a lower reliability level than that required for new structures, they may still comply with the performance requirements.

The requirements for safety and serviceability specified in the international standard (ISO 13822, 2001) are in principle the same as those recommended for design of new structures. There are, however, some fundamental differences between the criteria for design of new structures and assessment of existing structures indicated in Tab. 1. It is generally required to minimize structural intervention to existing structures and to use the existing materials. Actual properties of existing materials should be, however, carefully verified.

Table 1. Criteria for assessment of existing and design of new structures.

Criteria	Existing structures	New structures
Economical	incremental cost of increasing the structural safety is commonly high	incremental cost of increasing the structural safety is commonly lower
Social	may be significant due to reduction or disruption of serviceability and preservation of heritage values	commonly less significant than for existing structures
Sustainability	in large measure existing materials are used, leading to reduction of waste and recycling	commonly new materials are applied

2. Reliability verification of existing bridges

Verification of the serviceability limit states of an existing bridge is commonly based on estimation of the remaining working life. Current international standards give general recommendations only. Eurocode (EN

1990, 2002) for the basis of structural design gives indicative values of the design working life for several categories of structures (100 years for bridges).
Recently developed Czech standard (CSN 73 6222, 2008) provides basic guidance for determination of the load-bearing capacity and for estimation of the residual working life of existing concrete bridges. Six bridge categories of prestressed and reinforced concrete bridges are distinguished and limiting values of crack width are recommended.
For specification of the load-bearing capacity of a prestressed or reinforced concrete bridge in operating conditions, the limit states of decompression and crack width have to be verified. The procedure for the assessment of the remaining working life of an existing reinforced concrete bridge based on crack width limit as proposed in new documents (CSN 73 6222, 2008) and (Model Code, 2010) is analysed in detail. The indicative remaining working life of a concrete bridge is estimated on the basis of crack width limit given in Table 2.

Table 2. Remaining working life of concrete bridges based on crack width limit

Remaining working life (in years)	Post-tensioned concrete bridges with tendons		Reinforced concrete bridges
	bonded	non-bonded	
50	0,2 mm	0,2 mm	0,3 mm
25	0,2 mm	0,3 mm	0,4 mm
10	0,3 mm	0,4 mm	0,5 mm

3. Verification of crack width according to Eurocodes

The formula for the fully developed crack width as recommended in Eurocodes is based on the model provided in (CEB, 1985). The mean crack width w_m is given as

$$w_m = s_{r,m} (\varepsilon_{sm} - \varepsilon_{cm}) \tag{1}$$

where $s_{r,m}$ is the mean crack spacing, ε_{sm} is the mean strain in reinforcement under the relevant combination of actions and ε_{cm} is the mean strain in concrete between the cracks. The mean crack spacing is estimated as

$$s_{r,m} = kc + 0{,}25 k_1 k_2 d / \rho_{p,eff} \tag{2}$$

where c is the concrete cover of reinforcement, k is the coefficient for cover characteristics (k = 2), k_1 is the coefficient for bond properties of reinforcement (0,8 for high bond bars), k_2 is the coefficient for stress distribution (0,5 for bending), d is the bar diameter and $\rho_{p,eff}$ is the effective reinforcement ratio.

The strains difference $\varepsilon_{sm} - \varepsilon_{cm}$ is expressed as

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - (1+\alpha_e \rho_{p,eff})\, k_t \dfrac{f_{ct,eff}}{\rho_{p,eff}}}{E_s} \tag{3}$$

where σ_s is the stress in the tension reinforcement assuming a cracked section, α_e is the ratio E_s/E_{cm} between the modulus of steel and concrete and k_t is the factor dependent on the load duration (0,4 for long-term loading).

The characteristic crack width w_k (the 5% upper fractile) is estimated on the basis of the mean crack width w_m assuming a normal distribution as

$$w_k = (1 + u_p\, V)\, w_m \approx 1{,}7 w_m \tag{4}$$

where V is the coefficient of variation of the crack width (up to 40 %) and u_p is the 5% upper fractile of the standardized normal distribution. The relationship (4) is applied in the probabilistic reliability analysis as follows.

The Eurocode (EN 1992-1-1, 2004) provides the formula for the characteristic crack width in the form

$$w_k = s_{r,max}\, (\varepsilon_{sm} - \varepsilon_{cm}) = 1{,}7\, s_{r,m}\, (\varepsilon_{sm} - \varepsilon_{cm}) \tag{5}$$

For the verification of the limit states of crack width of a bridge cross-section, the following inequality has to be fulfilled

$$w_k \leq w_{lim} \tag{6}$$

where w_{lim} is the crack width limit. The Eurocodes recommend for reinforced concrete bridges the crack width limit w_{lim} = 0,3 mm under the quasi-permanent load combination.

4. Verification of crack width according to Model Code 2010

The document (Model Code, 2010) recommends a crack width model which is also based on formulae (4) and (5). The characteristic crack spacing is expressed as

$$s_{r,k} = 2(kc + \frac{f_{ctk,0,05}}{4\tau_{bk}} \frac{d}{\rho_{p,eff}}) \tag{7}$$

where k is the coefficient for cover characteristics, c is the reinforcement cover, d is the bar diameter and τ_{bk} is the lower fractile of average bond stress.

The strains difference $\varepsilon_{sm} - \varepsilon_{cm}$ is also given by expression (3) in which instead of the coefficient k_t a similar coefficient b_t for the effects of load duration is applied. The same crack width limit w_{lim} = 0,3 mm under the quasi-permanent load combination is recommended for reinforced concrete structures in (Model Code, 2010).

5. Reliability analysis of an existing bridge

A simply supported reinforced concrete bridge is considered as an example for estimation of the deterioration effects due to reinforcement corrosion on bridge reliability. The residual working life of a bridge is analyzed on the basis of the design criteria for crack width provided in documents (EN 1992-2, 2005), (Model Code, 2010) and (CSN 73 6222, 2008).

Span of the bridge is 15 m, the height of slab is 0,9 m and width of the road is 7,5 m. It is assumed that the bridge is loaded by self-weight, permanent actions and traffic loads. The traffic load model LM1 is considered as given in (EN 1991-2, 2005) consisting of the double-axle concentrated load $\alpha_Q Q_k$ (the tandem system TS, two lanes loaded by the axle loads 300 kN and 200 kN with the adjustment factor α_Q = 0,8, and the uniformly distributed load $\alpha_q q_k$ (UDL, the first lane q_1 = 9 kN/m^2 with adjustment factor α_q = 0,8, the second lane q_i = 2,5 kN/m^2 with factor α_{qi} = 1). The bridge is made of concrete class C 40/50 and reinforcement S 500.

The reinforced concrete bridge is designed according to (EN 1992-2, 2006) and the area of required reinforcement in the mid-span of the cross-section is determined as 0,0058 m^2. The design of the bridge fulfils the requirements of Eurocodes including the minimum area of reinforcement for limiting of crack width. The characteristic crack width w_k = 0,298 mm is specified according to (EN 1992-1-1, 2004) being less than the recommended crack width limit w_{lim} = 0,3 mm. In case that the new document MC 2010 (Model Code, 2010) is applied, the limiting crack width is also satisfied.

It is assumed that the bridge is gradually deteriorating. Two study cases of the uniform corrosion and pitting corrosion are considered according to the models given in both prescriptive documents. It is assumed that the initiation of corrosion starts early after the bridge completion.

The reduction of the reinforcement area due to the uniform and also pitting corrosion in time (years) is illustrated in Figure 1.

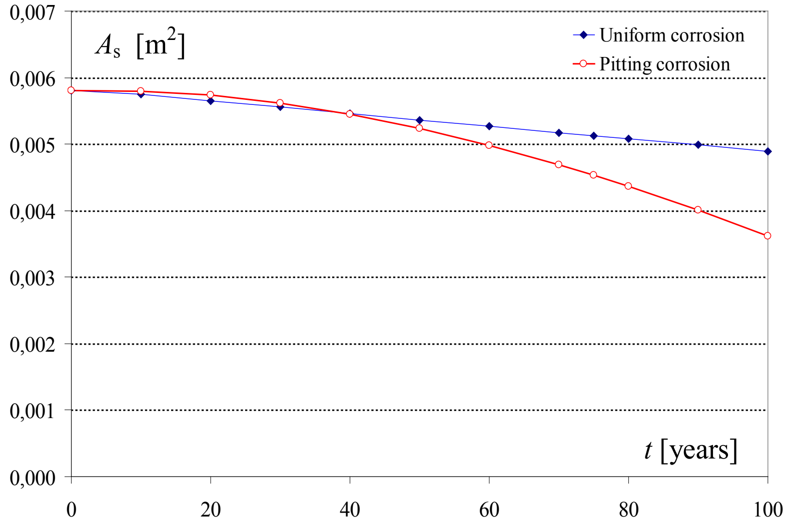

Fig. 1 Variation of the designed reinforcement area A_s with time t due to uniform and pitting corrosion.

6. Probabilistic reliability analysis

The probabilistic methods are applied for the verification of the reliability level of the existing bridge affected by corrosion with respect to the serviceability limit states of crack width. A homogeneous (uniform) corrosion and also localized (pitting) corrosion are considered, (Model Code, 2010), (Val et al. 1998).
The limit state function $g(.)$ is expressed in terms of the limit value of the crack width w_{lim} and the random crack width $w(.)$ calculated under the theoretical crack width models and quasi-permanent combination of actions given as

$$g(\boldsymbol{X},t) = \xi_{\text{lim}}\, w_{\text{lim}} - \xi_w\, w(\boldsymbol{X},t) \tag{8}$$

where $\boldsymbol{X}$ is the vector of basic variables, ξ_{lim} and ξ_w are the coefficients of model uncertainties for the requirements on the crack width limit and the crack width model, respectively, and t is the considered time.

The random crack width is considered by expression (1) in which the symbol $w(.)$ is used instead of w_{m}. The relationship between expressions (1) and (5) is taken into account in the reliability analysis as follows. The probability P_{F} of a random crack width $w(\boldsymbol{X},\, t)$ exceeding the crack width limit w_{lim} for the time dependent problem may be assessed as

$$P_{\mathrm{F}}(\boldsymbol{X}, t) = P\{\xi_{\mathrm{lim}}\, w_{\mathrm{lim}} - \xi_w\, w(\boldsymbol{X}, t) < 0\} \tag{9}$$

The bridge may be considered as reliable if the following inequality is satisfied

$$P_{\mathrm{F}}\,(\boldsymbol{X}, t) \leq P_{\mathrm{Ft}} \tag{10}$$

where the probability of failure P_{Ft} is the specified (target) value that should not be exceeded during the design working life. Another reliability indicator is the generalized reliability index β, defined on the basis of the probability of failure P_{F}, given as $\beta\,(\boldsymbol{X},\ t) = -\Phi^{-1}(P_{\mathrm{F}}(\boldsymbol{X},\ t))$. The target reliability index β_{t} for verification of the irreversible serviceability limit states is $\beta_{\mathrm{t}} = 1{,}5$.

The design of a bridge considered in the following study fulfils the requirements of (EN 1992-1-1, 2004) and (EN 1992-2, 2005) for the ultimate limit states and the minimum area of reinforcement needed for limiting of cracking.

The probabilistic models applied in the reliability analysis are listed in Table 3. Some of the basic variables entering expression (9) are assumed to be deterministic values denoted DET (reinforcement area, some geometric characteristic, coefficients k_{t} and k_2) while the others are considered as random variables having normal (N), lognormal (LN), Beta (BET) and Gumbel (GUM) distributions.

Table 3. Probabilistic models of basic variables.

Basic variable	Symbol	Distr.	Mean μ	St. dev. σ
Concrete tensile strength	f_{ct}	LN	3,5	0,7
Modulus of elasticity for steel	E_{s}	DET	200000	-
Concrete modulus of elasticity	E_{c}	LN	35000	3500
Creep coefficient	φ	LN	1,46	0,4
Coefficient of bond strength	k_1	LN	0,8	0,21
Coefficient for cover	k	LN	2	0,5
Length of span	L	DET	15	-
Diameter of bar	d	DET	0,028	-
Cross-section height	h	DET	0,9	-

Surfacing thickness	h_1	LN	0,1	0,01
Reinforcement cover	c	BET *	0,04	0,01
Crack width model uncertainty	ξ_w	LN	1,0	0,15μ
Crack width limit uncertainty	$\xi_{\lim}$	LN	1,0	0,1μ
Density of concrete	ρ	N	2500	0,08μ
Tandem system (TS)	Q	GUM	500	58
UDL system	q	GUM	20	0,2

* Lower bound 0, upper bound 2μ.

The probabilistic traffic load model is based on the traffic measurements on the motorway A6 near Auxerre which was selected for the development of the models of traffic actions, see (Hanswille, 2007). The probabilistic model of traffic loads based on the Gumbel distribution considered here takes into account the bridge remaining working life.

The results of probabilistic reliability analyses of the reinforced concrete bridge with respect to the limit states of crack width considering the prescriptive documents (EN 1992, 2005) and (MC, 2010) are illustrated in Figure 2. Two study cases of the uniform and pitting corrosion are taken into account.

The initial reliability of the bridge with respect to crack width (β = 2,1 considering EN 1992-2, β = 2,3 for MC 2010) is greater than the target value of reliability index β_t =1,5 recommended for verification of the serviceability limit states. However, the diminishing area of reinforcement due to reinforcement corrosion leads to the decrease of the reliability index β in time as it is shown in Figure 2.

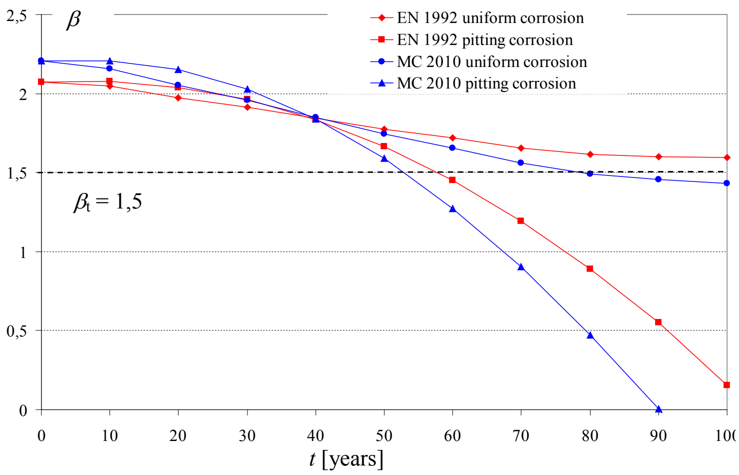

Fig. 2 Variation of the reliability index β for uniform and pitting corrosion with time t considering analytical models of crack width in new European documents.

The results of probabilistic analysis indicate that the reliability of a bridge affected by pitting corrosion after its 60 year working life significantly decreases below the target reliability level. The uniform corrosion also leads to reduction of the area of reinforcement, however having smaller impact to the bridge reliability in time. Thus, considering the degradation processes, for the achievement of the recommended target reliability level β_t = 1,5 during the whole working life of the bridge some further provisions need to be accepted in design, e.g. increase of reinforcement design area or acceptance of protective measures against corrosion.

In the following study case it is considered that the bridge fulfills the requirements of Eurocodes for the ultimate limit states and the minimum area of reinforcement needed for limiting cracking. The probabilistic reliability analyses indicates that the initial reliability index (β = 1,65) of the bridge with respect to crack width still fulfils the required target value (β_t = 1,5). However, the reliability of the bridge decreases in time due to the corrosion process. It is shown in Figure 3 that the reliability index decreases below the required reliability level after the first 30 years of the bridge working life.

Then, the decrease of the reliability caused by pitting corrosion is going on with considerably greater rate than in case of uniform corrosion. For the bridge working life from 50 to 75 years, the reliability index significantly decreases below the target value due to the reinforcement reduction caused by pitting corrosion (in 75 years up to β = 0,6).

In case that the higher crack width limit $w_{\lim}$ = 0,004 mm may be considered in 75 years of the bridge working life (remaining 25 years in Table 2) as recommended in the national provisions, the reliability index of the bridge affected by the pitting corrosion increases up to β = 1,5 meeting the required target value. However, the reliability of the bridge is significantly decreasing in the next time.

In case that 90 years of bridge working life (remaining 10 years) is assumed, the crack width limit $w_{\lim}$ = 0,005 may be applied. Then the reliability index increases up to β = 1,7 for the pitting corrosion and then again significantly decreases.

When the uniform corrosion of bridge reinforcement is considered, the reliability index decreases less than in the case of pitting corrosion, fulfilling the target value of reliability index till 30 years. The reliability index decreases from 1,35 to 1,2 for the time interval from 50 to 70 years of bridge working life. The crack width limit 0,004 mm, resp. 0,005 mm, allowed in (CSN 73 6222, 2008) for the bridge remaining life-time of 25 years, resp. 10 years, seems to be proposed rather high leading to high values of the reliability index.

It appears that the recommended values of crack width limits should take into account the character of deterioration process.

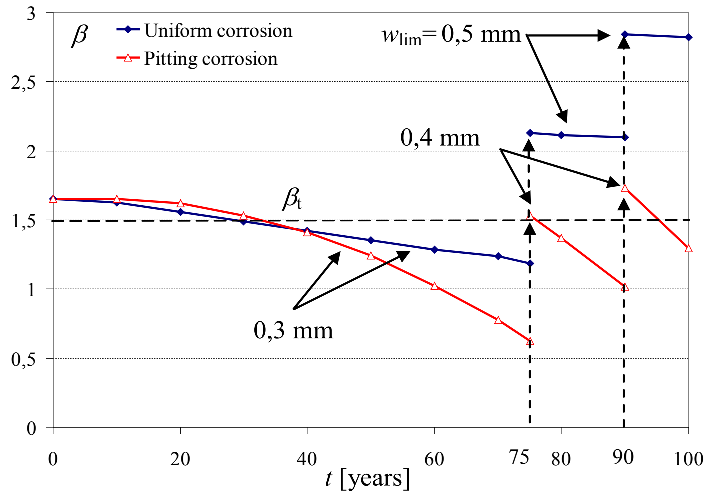

Fig. 3 Variation of the reliability index β for uniform and pitting corrosion in time t for recommended crack width limits w_{lim} connected to the remain working life of bridge.

The MC 2010 crack width model in comparison to EN 1992 model is more conservative, see Figure 4 for considered pitting corrosion.

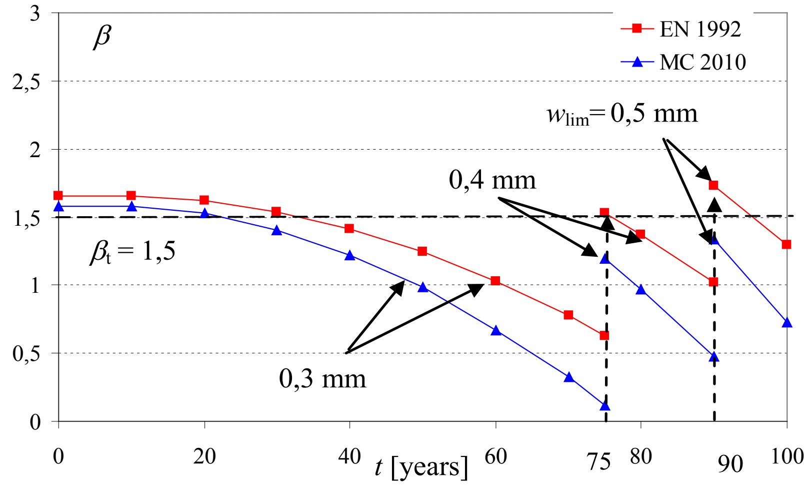

Fig. 4 Variation of the reliability index β for pitting corrosion in time t for recommended crack width limits w_{lim} and crack width models given in EN 1992 and MC 2010.

7. Concluding remarks

The reliability analysis of a reinforced concrete bridge regarding the serviceability limit states of crack width indicates that the uniform corrosion leads to a smaller reduction of the reinforcement area and higher reliability indices than the pitting corrosion.

The results of probabilistic analysis of a selected deteriorating bridge indicate that its reliability after first half of bridge working life may be rather low (β < 1,3). Thus, to achieve the recommended target reliability level during the whole working life of the bridge, additional provisions need to be accepted in the design (e.g. increase of reinforcement cover, acceptance of protective measures).

The crack width models provided in the new European documents leads to similar but slightly favourable results than MC 2010.

The serviceability constrains recommended for the assessment of the residual working life of a bridge in current prescriptive documents should be further
analyzed and calibrated. The type of corrosion (uniform, pitting) and potential consequences of failure should be taken into account. It appears that the probabilistic assessment of existing bridges may facilitate the optimum decision regarding their safety and serviceability, and indirectly contribute to a sustainable development.

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